3. Performance Evaluation

3.1 Scope

This chapter provides simplified criteria for evaluating the probable seismic performance of existing welded steel moment-frame buildings. These procedures may be used to quantify the ability of a building to achieve desired performance objectives, either before or after the construction of structural upgrades. It includes definition of performance objectives, discussions of expected performance of buildings conforming to *FEMA-302 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, and procedures for estimating a level of confidence that a building will provide a desired level of performance for specified earthquake hazards. It is applicable only to well configured, regular structures as defined in *FEMA-302*. A more detailed procedure, applicable to irregular structures and performance objectives based on deterministic earthquake scenarios is presented in Appendix A of these *Recommended Criteria*.

Commentary: These recommendations only address methods of evaluating structural performance of welded steel moment-frame buildings. Although the performance of nonstructural components of buildings is critically important to the way in which buildings are used following an earthquake, treatment of this topic is beyond the scope of these Recommended Criteria. FEMA-273 – NEHRP Guidelines for the Seismic Rehabilitation of Buildings provides a more complete set of recommendations with regard to evaluating the performance of nonstructural components.

FEMA-355F – State of the Art Report on Performance Prediction and Evaluation, presents, in detail, the basis for the procedures contained herein and the derivation of the various parameters used in the procedures.

3.2 Performance Definitions

The performance evaluation procedures contained in these *Recommended Criteria* permit estimation of a level of confidence that a structure will be able to achieve a desired performance objective. Each performance objective consists of the specification of a structural performance level and a corresponding hazard level, for which that performance level is to be achieved. For example, a seismic upgrade design may be intended to provide a 95% level of confidence that a structure provide Collapse Prevention or better performance for earthquake hazards with a 2% probability of exceedance in 50 years, or a 50% confidence level that a structure provide Immediate Occupancy or better performance, for earthquake hazards with a 50% probability of exceedance in 50 years. The user may determine the level of confidence associated with achieving any desired performance objective.

Commentary: The performance evaluation procedures contained in these Recommended Criteria are based on an approach first developed in FEMA-273. However, substantial modifications have been made to the procedures presented in that document.

In FEMA-273, performance objectives are expressed in a deterministic manner. Each performance objective consists of the specification of a limiting damage state, termed a performance level, together with a specification of the ground motion intensity for which that (or better) performance is to be provided. This implies a warranty that if the specified ground motion is actually experienced by a building designed using the FEMA-273 procedures, damage will be no worse than that indicated in the performance objective.

In reality, it is very difficult to predict with certainty how much damage a building will experience for a given level of ground motion. This is because there are many factors that affect the behavior and response of a building, such as the stiffness of nonstructural elements, the strength of individual building components, and the quality of construction, that can not be precisely defined and also, because the analysis procedures used to predict building response are not completely accurate. In addition, the exact character of the ground motion that will actually affect a building is itself very uncertain. Given these uncertainties, it is inappropriate to imply that performance can be predicted in an absolute sense, and correspondingly, that it is absolutely possible to produce designs that will achieve desired performance objectives.

In recognition of this, these Recommended Criteria adopt a reliability-based probabilistic approach to performance evaluation that explicitly acknowledges these inherent uncertainties. These uncertainties are expressed in terms of a confidence level. If an evaluation indicates a high level of confidence, for example 90 or 95% that a performance objective can be achieved, then this means it is very likely (but not guaranteed) that the building will be capable of meeting the desired performance. If lower confidence is calculated, for example 50%, this is an indication that the building may not be capable of meeting the desired performance objective. If still lower confidence is calculated, for example 30%, then this indicates the building will likely not be able to meet the desired performance objective. Increased confidence in a building's ability to provide specific performance can be obtained in three basic ways.

- *Providing the building with greater earthquake resistance, for example, by designing the structure to be stiffer and stronger*
- *Reducing some of the uncertainty inherent in the performance evaluation process through the use of more accurate structural models and analyses and better data on the building's configuration, strength and stiffness.*
- *More accurately characterizing the uncertainties inherent in the performance evaluation process, for example, by using the more exact procedures of Appendix A of these Recommended Criteria.*

Refer also to the commentary in Section 3.2.1.2 for additional discussion of the probabilistic approach adopted by these Recommended Criteria.

3.2.1 Hazards

3.2.1.1 General

Earthquake hazards include direct ground fault rupture, ground shaking, liquefaction, lateral spreading, and land sliding. Of these various potential hazards, the one that effects the largest number of structures and causes the most widespread damage is ground shaking. Ground shaking is the only earthquake hazard that the structural design provisions of the building codes directly address. However, for structures located on sites where any of the other hazards can result in significant ground deformation, these hazards should also be considered in a structural performance evaluation.

3.2.1.2 Ground Shaking

Ground shaking hazards are typically characterized by a hazard curve, which indicates the probability that a given value of a ground motion parameter, for example peak ground acceleration, will be exceeded over a certain period of time, and by acceleration response spectra or ground motion accelerograms that are compatible with the values of the ground motion parameters obtained from the hazard curve and the local site geology. The ground shaking hazard maps provided with the *FEMA-302 NEHRP Recommended Provisions* and the *FEMA-273 NEHRP Rehabilitation Guidelines* have been prepared based on hazard curves that have been developed by the United States Geological Survey for a grid-work of sites encompassing the United States and its territories. *FEMA-302* defines two specific levels of hazard for consideration in design and specifies methods for developing response spectra for each of these levels. The two levels are:

1. Maximum Considered Earthquake (MCE) ground shaking. This is the most severe level of ground shaking that is deemed appropriate for consideration in the design process for building structures, though not necessarily the most severe level of ground shaking that could ever be experienced at a site. In most regions, this ground shaking has a 2% probability of exceedance in 50 years, or roughly a 2,500 year mean recurrence interval. In regions of very high seismicity, near major active faults, the MCE ground shaking level is limited by a conservative, deterministic estimate of the ground shaking resulting from a maximum magnitude earthquake on the known active faults in the region. The probability that such deterministic ground shaking will be experienced at a site can vary considerably, depending on the activity rate of the individual fault. Refer to *FEMA-303, Commentary to the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* for more detailed information on this issue.

2. Design Earthquake (DE) ground shaking. This is the ground shaking level upon which design lateral forces, used as the basis for analysis and design in *FEMA-302*, are based. It is defined as a spectrum that is 2/3 of the shaking intensity calculated for the MCE spectrum, at each period. The probability that DE ground shaking will be experienced varies, depending on the regional, and, in some cases, site, seismicity.

> *Commentary: The mean recurrence interval for Design Earthquake ground shaking will vary depending on regional and site seismicity. In areas of low*

seismicity the hazard return period will generally range between 750-1,250 years and will remain relatively constant across neighboring communities. In areas of high seismicity the recurrence interval may range between 300-600 years and can vary significantly within a distance of a few miles.

Performance evaluation conducted in accordance with these *Recommended Criteria* may be performed for any level of ground shaking. Ground shaking will typically be determined probabilistically, i.e., based on the probability that shaking of the specified intensity will be experienced at a site. Ground shaking must be characterized by an acceleration response spectrum or a suite of ground motion accelerograms compatible with that spectrum. In addition, a coefficient *k* that relates the rate of change in ground motion intensity with change in probability, is required. *FEMA-273* provides guidelines for development of ground motion response spectra at different probabilities of exceedance. The procedures of this chapter use a default value for the coefficient *k*, as described in the commentary to Section 3.6. Performance evaluation for deterministic ground motion based on specific earthquake scenarios, for example an earthquake of given magnitude on a specific fault can also be performed. Appendix A provides procedures that may be used for deterministically defined hazards.

Commentary: Detailed guidelines on ground-motion estimation and characterization are beyond the scope of this publication. Those interested in such information are referred to FEMA-303 and FEMA 274 Commentary to the NEHRP Guidelines for Seismic Rehabilitation of Buildings and references noted therein.

Although Section 3.2 of these Recommended Criteria indicates that performance objectives are an expression of the desired performance for a building, given that ground motion of certain intensity is experienced, this is a significant simplification. In reality, the performance objectives are statements of the total probability that damage experienced by a building in a period of years will be more severe than the desired amount (performance level), given our knowledge of the site seismicity. Although it is transparent to the user, this is obtained by integrating the conditional probability that building response exceeds the limiting response for a performance level, given a ground motion intensity, over the probability of experiencing different intensities of ground motion, as represented by the site hazard curve, and specifically, the coefficient k which is the logarithmic slope of the hazard curve, at the desired hazard level. Thus, a performance objective that is stated as "meeting collapse prevention performance for ground shaking with a 2% probability of exceedance in 50 years" should more correctly be stated as being "less than a 2% chance in 50 years of damage more severe than the collapse prevention level, given the mean definition of seismicity."

The procedures contained in this chapter neglect uncertainties associated with the definition of the seismicity, that is, the intensity of ground shaking at various probabilities. Such uncertainties can be as large, and perhaps larger, than the uncertainties associated with structural performance estimation. Thus the confidence calculated in accordance with the procedures of this chapter is really a confidence associate with structural performance, given the presumed seismicity.

The simplified procedures presented in this chapter have been developed using hazard parameters typical of coastal California. They can be conservatively applied in regions of lower seismicity without the need to determine site specific hazard parameters. However, accurate definition of the hazard is a critical part of the performance evaluation procedures contained herein and in regions of lower seismicity, may result in calculation of higher confidence. Appendix A of these Recommended Criteria presents more detailed procedures that may be used to consider directly the site-specific characteristics of hazard in the evaluation of performance.

3.2.1.3 Other Hazards

In order to predict reliably the probable performance of a structure, it is necessary to determine if earthquake hazards other than ground shaking, including direct ground fault rupture, liquefaction, lateral spreading, and land sliding are likely to occur at a site and to estimate the severity of these effects. The severity of ground fault rupture, lateral spreading and land sliding is characterized by an estimate of permanent ground deformation. The severity of liquefaction is characterized by an estimate of the potential loss in bearing strength of subsoil layers and permanent ground settlement. In order to determine the performance of a structure which is subject to these hazards, the effects of the projected ground displacements should be evaluated using a mathematical model of the structure. The severity of these hazards (i.e. probability of exceedance) used in performance evaluation should be compatible with that used in specification of ground shaking hazards.

Commentary: Most sites are not at significant risk from earthquake hazards other than ground shaking. However, these hazards can be very destructive to structures located on sites where they occur. Accurate determination of the propensity of a site to experience these hazards requires site-specific study by a competent earth scientist or geotechnical engineer. Guidelines on such assessments are beyond the scope of this publication.

3.2.2 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 3-1 describes the overall levels of structural and nonstructural damage that may be expected of buildings meeting two performance levels, termed Collapse Prevention and Immediate Occupancy. These performance descriptions are not precise and variation among buildings must be expected within the same Performance Level. The structural performance levels are presented in Section 3.2.2.2.

Table 3-1 Building Performance Levels

	Building Performance Levels	
	Collapse Prevention Level	**Immediate Occupancy Level**
Overall Damage	Severe	Light
General	Little residual stiffness and strength, but gravity loads are supported. Large permanent drifts. Some exits may be blocked. Exterior cladding may be extensively damaged and some local failures may occur. Building is near collapse.	Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, ceilings, and structural elements. Elevators can be restarted. Fire protection operable.
Nonstructural components	Extensive damage.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.
Comparison with performance intended by *FEMA-302* for SUG-I buildings when subjected to the Design Earthquake	Significantly more damage and greater risk.	Much less damage and lower risk.
Comparison with performance intended by *FEMA-302* for SUG-I buildings when subjected to the Maximum Considered Earthquake	Same level of performance	Much less damage and lower risk.
SUG = Seismic Use Group		

Commentary: Building performance is expressed in terms of building performance levels. These building performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that WMSF buildings could experience as a result of earthquake response. The particular damage states identified as building performance levels have been selected because these performance levels have readily identifiable consequences associated with the postearthquake disposition of the building that are meaningful to the building user community and also because they are quantifiable in technical terms. These include the ability to resume normal functions within the building, the advisability of postearthquake occupancy, and the risk to life safety.

Although a building's performance is a function of the performance of both structural systems and nonstructural components and contents, only the structural performance levels are defined in these Recommended Criteria. The reference to nonstructural components above is to remind the reader of the probable performance of these elements at the various performance levels.

3.2.2.1 Nonstructural Performance Levels

These *Recommended Criteria* only addresses methods of evaluating structural performance of steel moment-frame buildings. Although the performance of nonstructural components of buildings are critically important to the way in which buildings are used following an earthquake, treatment of this topic is beyond the scope of these *Recommended Criteria*. *FEMA-273* provides a more complete set of recommendations with regard to evaluating the performance of nonstructural components.

3.2.2.2 Structural Performance Levels

Two discrete structural performance levels, Collapse Prevention and Immediate Occupancy are defined in these *Recommended Criteria*. Table 3-2 relates these structural performance levels to the limiting damage states for framing elements of steel moment-frame structures. Acceptance criteria, which relate to the permissible interstory drifts and earthquake-induced forces for the various elements of steel moment-frame structures, are tied directly to these structural performance levels and are presented in later sections of these *Recommended Criteria*.

Commentary: FEMA-273 defines three structural performance levels, Immediate Occupancy, Life Safety and Collapse Prevention and also defines two performance ranges. These performance ranges, rather than representing discrete damage states, span the entire spectrum of potential damage states between no damage and total damage. No acceptance criteria are provided for these performance ranges in FEMA-273. Rather, these must be determined on a project-specific basis, by interpolation or extrapolation from the criteria provided for the three performance levels. Performance ranges, as such, are not defined in these Recommended Criteria. However, compatible with the FEMA-273 approach, users have the ability to create their own, custom performance levels, and to develop acceptance criteria for these levels, based on interpolation between the two performance levels, to suit the needs of a specific project. When such interpolation is performed, it is not possible to associate a confidence level with achievement of these intermediate performance definitions.

The Life Safety performance level contained in FEMA-273 and FEMA-302 is not included in these Recommended Criteria. As defined in FEMA-273 and FEMA-302, the Life Safety level is a damage state in which significant damage has been sustained, although some margin remains against either partial or total collapse. In FEMA-273 this margin is taken as 1/3. That is, it is anticipated that a ground motion level that is 1/3 larger than that which results in the Life Safety performance level for a building would be required to bring the building to the

Collapse Prevention level. In FEMA-302, this margin is taken as ½, i.e. it is believed that buildings designed for Life Safety performance can experience approximately 50% greater motion before they reach the Collapse Prevention level. Due to the somewhat arbitrary definition of this performance level, and the fact that different guidelines and codes have selected alternative definitions for it (as described above), the Life Safety level has not been included in these Recommended Criteria. However, as with the performance ranges, users desiring to evaluate buildings for the Life Safety performance level may do so by interpolating between the acceptance criteria provided for the Collapse Prevention and Immediate Occupancy levels.

Table 3-2 Structural Performance Levels

		Structural Performance Levels	
Elements	**Type**	**Collapse Prevention**	**Immediate Occupancy**
Girder		Extensive distortion, local yielding and buckling. A few girders may experience partial fractures	Minor local yielding and buckling at a few places.
Column		Moderate distortion; some columns experience yielding. Some local buckling of flanges	No observable damage or distortion
Beam-Column Connections	Connection Type 1[1]	Some fractures with some connections experiencing near total loss of capacity	Less than 10% of connections fractured on any one floor; minor yielding at other connections
	Connection Type 2[1]	Many fractures with some connections experiencing near total loss of capacity	Less than 10% of connections fractured on any one floor; minor yielding at other connections
Panel Zone		Extensive distortion	Minor distortion
Column Splice		No fractures	No yielding
Base Plate		Extensive yielding of anchor bolts and base plate	No observable damage or distortion
Drift	Interstory	Large permanent	Less than 1% permanent

Notes: 1 Connection types are defined in Section 3.6.2.1, Table 3-9.

3.2.2.2.1 Collapse Prevention Performance Level

The Collapse Prevention structural performance level is defined as the postearthquake damage state in which the structure is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and, to a more limited extent, degradation in the vertical load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity-load demands. The structure may not be technically or economically practical to repair and is not safe for re-occupancy; aftershock activity could credibly induce collapse.

3.2.2.2.2 Immediate Occupancy Performance Level

The Immediate Occupancy structural performance level is defined as the postearthquake damage state in which only limited structural damage has occurred. Damage is anticipated to be so slight that it would not be necessary to inspect the building for damage following the earthquake, and such little damage as may be present would not require repair. The basic vertical- and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low. Buildings meeting this performance level should be safe for immediate postearthquake occupancy, presuming that damage to nonstructural components is suitably light and that needed utility services are available.

Commentary: When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced in the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within their elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.

At more severe levels of ground motion, the lateral deformations induced in the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of buckling, yielding and fracturing of the various components. As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage may be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to *collapse of the structural system. At higher levels of ground motion, the lateral deformations induced in a structure will (1), strain a number of elements to a point at which the elements degrade in stiffness and strength, or (2), as a result of P-D effects, the structure loses stability. Eventually, partial or total collapse of the structure can occur.*

The structural performance levels relate the extent of a building's response to earthquake hazards to these various possible damage states. At the Immediate Occupancy Level, degradation of strength and stiffness in beam-column connections is limited to approximately 10% of the connections on any given floor and throughout the structure as a whole. The structure retains a significant portion of its original stiffness and most, if not all, of its strength, although some slight permanent drift may result. At the Collapse Prevention level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and can collapse.

3.3 Evaluation Approach

The basic process of performance evaluation, as contained in these *Recommended Criteria* is to develop a mathematical model of the structure and to evaluate its response to the earthquake hazard by one or more methods of structural analysis. The structural analysis is used to predict the value of various structural response parameters. These include:

- interstory drift, and
- axial forces on individual columns.

These structural response parameters are related to the amount of damage experienced by individual structural components as well as to the structure as a whole. For each performance level, these *Recommended Criteria* specify acceptance criteria (median estimates of capacity) for all the design parameters indicated above. Acceptability of structural performance is evaluated considering both local performance (at the element level) and global performance. Acceptance criteria have been developed on a reliability basis, incorporating demand and resistance factors related to the uncertainty inherent in the evaluation process and incorporating the variation inherent in structural response, such that a confidence level can be established with regard to the ability of a structure to provide specific performance at selected, low, probabilities of exceedance.

Once an analysis is performed, predicted demands are adjusted by two factors, an analytical uncertainty factor g_a, which corrects the analytically predicted demands for bias and uncertainty inherent in the analytical technique, and a demand variability factor, g, which accounts for other sources of variability in structural response. These predicted demands are compared against acceptance criteria, which have been modified by resistance factors f to account for uncertainties and variation inherent in structural capacity prediction. Procedures are given to calculate the level of confidence provided by a seismic evaluation or upgrade design, to achieve a specific performance objective, based on the ratio of factored demand to factored capacity. If the predicted level of confidence is inadequate, then either more detailed investigations and analyses should be performed to improve the level of confidence attained with regard to performance, through the attainment of better understanding of the structure's behavior and modification of the demand and resistance factors, or the structure should be upgraded such that a sufficient level of confidence can be attained given the level of understanding. If it is deemed appropriate to upgrade a structure to improve its probable performance, an iterative approach consisting of trial design, followed by verification analysis, evaluation of design parameters against acceptance criteria, and calculation of confidence level is repeated until an acceptable upgrade design solution is found. Procedures for estimating confidence are contained in Section 3.6.

Commentary: These procedures adopt a demand and resistance factor design (DRFD) model for performance evaluation. This approach is similar to the Load and Resistance Factor design approach adopted by AISC LRFD except that the LRFD provisions are conducted on an element basis, rather than a structural system basis, and demands in these procedures can be drifts as well as forces and stresses. The purpose of this DRFD approach is to allow characterization of the confidence level inherent in a design with regard to a specific performance objective.

The factored interstory drift demand, calculated from the analysis represents a median estimate of the probable maximum interstory drift demand, at the desired probability of exceedance. Tables in these Recommended Criteria provide interstory drift capacities for the two performance levels for regular, well configured structures, dependent on structural system and connection type, as well as resistance factors f, that adjust the estimated capacity of the structure to median values. Appendix A provides procedures for determination of f factors for connections for which project-specific qualification testing is performed and a procedure that may be used to determine interstory drift capacities for irregular structures.

Once the factored demands and capacities are determined, an index parameter l is calculated from the ratio of the factored demands and capacities as indicated in Section 3.6. The value of l is then used to determine an associated confidence level based on tabulated values related to the uncertainty inherent in the estimation of the building's demand and capacities.

3.4 Analysis

In order to evaluate the performance of a welded steel moment-frame building it is necessary to construct a mathematical model of the structure that represents its strength and deformation characteristics, and to conduct an analysis to predict the values of various design parameters when it is subjected to design ground motion. This section provides guidelines for selecting an appropriate analysis procedure and for modeling. General requirements for the mathematical model are presented in Section 3.5.

3.4.1 Alternative Procedures

Four alternative analytical procedures are available for use in performance evaluation of welded steel moment-frame buildings. The basic analytical procedures are described in detail in *FEMA-273*. This section provides supplementary guidelines on the applicability of the *FEMA-273* procedures and also provides supplemental modeling recommendations. The four procedures are:

- linear static procedure – an equivalent lateral force technique, similar, but not identical, to that contained in many model building code provisions,
- linear dynamic procedure – an elastic, modal, response-spectrum analysis,
- nonlinear static procedure – a simplified nonlinear analysis procedure in which the forces and deformations induced by a monotonically increasing lateral loading is evaluated using a series of incremental elastic analyses of structures that are sequentially degraded to represent the effects of structural nonlinearity,
- nonlinear dynamic procedure – a nonlinear dynamic analysis procedure in which the response of a structure to a suite of ground motion histories is determined through numerical integration of the equations of motion for the structure. Structural stiffness is altered during the analysis to conform to nonlinear hysteretic models of the structural components.

Commentary: The purpose of structural analyses performed as part of the performance evaluation process is to predict the values of key response parameters that are indicative of the structure's performance when it is subjected to ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy (the appropriate level of confidence of achieving the desired performance) using the basic approach outlined in Section 3.6.

Analyses performed in support of design, as required by FEMA-302, evaluate the strength and deformation of the structure when it is subjected to a somewhat arbitrary level of loading. The loading is based on, but substantially reduced from, that predicted by an elastic analysis of the structure's dynamic response to the expected ground motions. Specifically, the loading is reduced by a factor R to account approximately for the beneficial effects of inelastic response.

Analyses conducted in support of performance evaluation, under these Recommended Criteria, take a markedly different approach. Rather than evaluating the forces and deformations induced in the structure under arbitrarily reduced loading levels, these analysis procedures attempt to predict, within probabilistically defined bounds, the actual values of the important response parameters in response to design ground motion.

The ability of the performance evaluation to estimate reliably the probable performance of the structure is dependent on the ability of the analysis procedure *to predict the values of these response parameters within acceptable levels of confidence. The linear dynamic procedure is able to provide relatively reliable estimates of the response parameters for structures that exhibit elastic, or near elastic, behavior. The linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it accounts less accurately for the dynamic characteristics of the building. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for buildings that exhibit significant nonlinear behavior, particularly if the buildings are irregular. However, it does not accurately account for the effects of higher mode response. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the building in response to ground motion. However, there are considerable uncertainties associated with the values of the response parameters predicted by this technique.*

3.4.2 Procedure Selection

Table 3-3 indicates the recommended analysis procedures for various performance levels and conditions of structural regularity.

3.4.3 Linear Static Procedure

3.4.3.1 Basis of the Procedure

Linear static procedure (LSP) analysis of steel moment-frame structures should be conducted in accordance with the recommendations of *FEMA-273*, except as noted herein. In this procedure, lateral forces are applied to the masses of the structure, and deflections and component forces under this applied loading are determined. Calculated internal forces typically will exceed those that the building can develop, because anticipated inelastic response of components and elements is not directly recognized by the procedure. The predicted interstory drifts and column axial forces are evaluated using the procedures of Section 3.6.

Table 3-3 Analysis Procedure Selection Criteria

Structural Characteristics				Analytical Procedure			
Performance Level	Fundamental Period, T	Regularity	Ratio of Column to Beam Strength	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Immediate Occupancy	$T \leq 3.5T_s$ [1]	Regular or Irregular	Any Conditions	Permitted	Permitted	Permitted	Permitted
	$T > 3.5\ T_s$ [1]	Regular or Irregular	Any Conditions	Not Permitted	Permitted	Not Permitted	Permitted
Collapse Prevention	$T \leq 3.5T_s$ [1]	Regular[2]	Strong Column[3]	Permitted	Permitted	Permitted	Permitted
			Weak Column[3]	Not Permitted	Not Permitted	Permitted	Permitted
		Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Permitted	Permitted
	$T > 3.5T_s$	Regular	Strong Column[3]	Not Permitted	Permitted	Not Permitted	Permitted
			Weak Column[3]	Not Permitted	Not Permitted	Not Permitted	Permitted
		Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Not Permitted	Permitted

Notes:

1- T_s is the period at which the response spectrum transitions from a domain of constant response acceleration (the plateau of the response spectrum curve) to one of constant spectral velocity. Refer to *FEMA-273* or *FEMA-302* for more information

2- Conditions of regularity are as defined in *FEMA-273*. These conditions are significantly different than those defined in *FEMA-302*.

3- A structure qualifies as having a strong column condition if at every floor level, the quantity $\Sigma M_{prc} / \Sigma M_{prb}$ is greater than 1.0, where ΣM_{prc} and ΣM_{prb} are the sum of the expected plastic moment strengths of the columns and beams that participate in the moment-resisting framing in a given direction of structural response.

Commentary: The linear static procedure is a method of estimating the response of the structure to earthquake ground shaking by representing the effects of this response through the application of a series of static lateral forces applied to an elastic mathematical model of the structure and its stiffness. The forces are applied to the structure in a pattern that represents the typical distribution of inertial forces in a regular structure responding in a linear manner to the ground shaking excitation, factored to account, in an approximate manner, for the probable inelastic behavior of the structure. It is assumed that the building response is dominated by the fundamental mode and that the lateral drifts induced, in the elastic structural model, by these forces represent a reasonable estimate of the actual deformation of the building when responding inelastically.

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. The static lateral forces, whose sum is equal to the pseudo lateral load, (so named in FEMA-273) represent earthquake demands for the LSP. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. However, if the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the yielding building will be less than the internal forces calculated on an elastic basis, but the predicted interstory drifts will approximate those that would actually occur in the structure.

The performance of welded steel moment-frame buildings is most closely related to the total inelastic deformation demands on the various seismic elements that comprise the structure, such as plastic rotation demands on beam-column assemblies and tensile demands on column splices. Linear analysis methods do not permit direct evaluation of such demands. However, through a series of analytical evaluations of typical buildings for a number of earthquake records, it has been possible to develop statistical correlation between the interstory drift demands predicted by a linear analysis and the actual inelastic deformation demands determined by more accurate nonlinear methods. These correlation relationships are reasonably valid for regular buildings, using the definitions of regularity in FEMA-273.

Although performance of welded steel moment-frame buildings is closely related to interstory drift demand, there are some failure mechanisms, notably, the failure of column splices, that are more closely related to strength demand. However, since inelastic structural behavior affects the strength demand on such elements, linear analysis is not capable of directly predicting these demands, *except when the structural response is essentially elastic. Therefore, when linear static analysis is performed for structures that respond in an inelastic manner, column axial demands should be estimated using a supplementary plastic analysis approach.*

The LSP is based on the assumption that the distribution of deformations predicted by an elastic analysis where all members remain linear elastic during all loadings, is similar to the distribution of deformations that will occur in actual nonlinear response. This assumption is inaccurate and can become more so for buildings that are highly irregular, that have response dominated by higher modes, or that experience large inelastic demands. It is for these reasons that alternative methods of analysis are recommended for irregular buildings and buildings with relatively long fundamental periods of vibration.

3.4.3.2 Period Determination

The fundamental period for each of the two orthogonal directions of building response shall be calculated by one of the following three methods.

Method 1. Eigenvalue (dynamic) analysis of the mathematical model of the building. The model for buildings with flexible diaphragms shall consider representation of diaphragm flexibility unless it can be shown that the effects of omission will not be significant.

Method 2. Evaluation of the following equation:

$$T = C_t h_n^{0.8} \tag{3-1}$$

where

T = fundamental period (in seconds) in the direction under consideration,
C_t = 0.028 for steel moment frames,
h_n = height (in feet) of the roof level above the base.

Method 3. The fundamental period of a one-story building with a single-span, flexible diaphragm may be calculated as:

$$T = (0.1D_w + 0.078D_d)^{0.5} \tag{3-2}$$

where D_w and D_d are in-plane frame and diaphragm displacements, respectively, in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm. For multiple-span diaphragms, a lateral load equal to the gravity weight tributary to the diaphragm span under consideration should be applied to each diaphragm span to calculate a separate period for each diaphragm span. The loads from each diaphragm should then be distributed to the frames using tributary load assumptions.

Commentary: The approximate period formula indicated in Method 2 is different from that contained in either FEMA-273 or FEMA-302. This formula has been adapted from recent study of the statistical distribution of measured periods in buildings obtained from accelerometer recordings of excitation occurring in past earthquakes (Goel and Chopra, 1997). This formula is intended to provide approximately an 84% confidence level (mean+ 1 s) that the actual period will exceed the calculated value. The formula has intentionally been selected to underestimate the actual period of the building as this will result in a conservatively large estimate of the calculated pseudo lateral force applied to the structure as a loading (See Section 3.4.3.3.1). The large pseudo lateral force will result in conservatively large estimates of interstory drift.

Use of the more accurate Method 1 procedure will typically result in lower estimates of interstory drift, and therefore increased confidence in the ability of a building to meet performance goals.

3.4.3.3 Determination of Actions and Deformations

3.4.3.3.1 Pseudo Lateral Load

The pseudo lateral load, given by Equation 3-3, shall be independently calculated for each of the two orthogonal directions of building response, and applied to a mathematical model of the structure.

$$V = C_1 C_2 C_3 S_a W \tag{3-3}$$

where:

C_1 = modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3 in *FEMA-273* with the elastic base shear capacity substituted for V_y. Alternatively, C_1 may be taken as having a value of 1.0 when the fundamental period T of the building response is greater than T_s and shall be taken as having a value of 1.5 when the fundamental period of the structure is equal to or less than T_0. Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T_0 = period at which the acceleration response spectrum for the site reaches its peak value, as indicated in *FEMA-302*. It may be taken as $0.2T_s$.

T_S = the characteristic period of the response spectrum, defined as the period associated with the transition from the constant spectral acceleration response segment of the spectrum to the constant spectral velocity response segment of the spectrum, as defined in *FEMA-302*.

C_2 = a modification factor to represent the effect of hysteretic pinching on the maximum displacement response. For steel moment-frame structures the value of C_2 shall be taken as 1.0.

C_3 = modification factor to represent increased dynamic displacements due to *P-D* effects and stiffness degradation. C_3 may be taken from Table 3-4 or shall be calculated from the equation:

$$C_3 = 1 + \frac{5(q_i - 0.1)}{T} \geq 1.0 \tag{3-4}$$

where:

q_i = the coefficient determined in accordance with Section 3.2.5.1 of *FEMA-273*.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration, for the hazard level corresponding to the performance objective being evaluated (i.e., probability of exceedance). The value of S_a may be calculated using the procedure outlined in Section 2.6.1.5 of *FEMA-273*.

W = Total dead load and anticipated live load as indicated below:

- in storage and warehouse occupancies, a minimum of 25% of the floor live load,
- the actual partition weight or minimum weight of 10 psf of floor area, whichever is greater,
- the applicable snow load – see the *NEHRP Recommended Provisions*,
- the total weight of permanent equipment and furnishings.

Commentary: The pseudo lateral force, when distributed over the height of the linearly-elastic model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation (3-3) may be significantly larger than the actual strength of the structure to resist this force. The acceptance evaluation procedures in Section 3.6 are developed to take this into account.

The values of the C_3 coefficient contained in Table 3-4 are conservative for most structures, and will generally result in calculation of an unduly low level of confidence. Use of Equation 3-4 to calculate C_3 is one way to improve calculated confidence without extensive additional effort, and is recommended.

Table 3-4 Modification Factors C_3 for Linear Static Procedure

Performance Level	C_3
Immediate Occupancy	1.0
Collapse Prevention	
Type 1[1] FR connections	1.2
Type 2[2] FR connections	1.4

Notes:
1. Type 1 connections are capable of resisting median total drift angle demands of 0.04 radians without fracture or strength degradation.
2. Type 2 connections are capable of resisting median total drift angle demands of 0.01 radians without fracture or strength degradation. Generally, welded unreinforced connections, employing weld metal with low notch toughness, typical of older steel moment-frame buildings should be considered to be of this type.

3.4.3.3.2 Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level x shall be determined as in Section 3.3.1.3B of *FEMA-273*.

3.4.3.3.3 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

3.4.3.3.4 Diaphragms

Floor and roof diaphragms shall be evaluated using the procedure outlined in Section 3.3.1.3D in *FEMA-273*. The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

3.4.3.3.5 Determination of Interstory Drift

Interstory drifts shall be calculated using lateral loads calculated in accordance with Section 3.4.3.3.1 and stiffness obtained from Section 3.5. Factored interstory drift demands $g_a g d_i$ at each story i, shall be determined by applying the appropriate analysis uncertainty factor g_a and demand variability factor g obtained from Section 3.6.2.

3.4.3.3.6 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces by the applicable analysis uncertainty factor γ_a and demand variability factor γ obtained in Section 3.6.3. Column forces shall be calculated either as:

1. the axial demands from the unreduced linear analysis, or
2. the axial demands computed from the equation:

$$P'_c = \pm\left[2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L - 2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R\right] \tag{3-5}$$

where:

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length L, of all moment-connected beams framing into the left hand side of the column, above the level under consideration, and

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length L, of all moment-connected beams framing into the right hand side of the column, above the level under consideration.

When a column is part of framing that resists lateral forces under multiple directions of loading, the seismic demand shall be taken as the most severe condition resulting from application of 100% of the seismic demand computed for any one direction of response with 30% of the seismic demand computed for the orthogonal direction of response.

3.4.4 Linear Dynamic Procedure

3.4.4.1 Basis of the Procedure

Linear dynamic procedure (LDP) analysis of steel moment frames shall be conducted in accordance with the recommendations in Section 3.3.2 of *FEMA-273* except as specifically noted herein. Coefficients C_1, C_2, and C_3 should be taken as indicated in Section 3.4.3.3 of these *Recommended Criteria*.

Estimates of interstory drift and column axial demands shall be evaluated using the applicable procedures of Section 3.6. Calculated displacements and column axial demands are factored by the applicable analytical uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6 and compared with factored capacity values for the appropriate performance level. Calculated internal forces typically will exceed those that the building can sustain because of inelastic response of components and elements, but are generally not used to evaluate performance.

Commentary: The linear dynamic procedure is similar in approach to the linear static procedure, described in Section 3.4.3. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, it use introduces somewhat less uncertainty than does the LSP. Coefficients C_1, C_2, and C_3, which account in an approximate manner for the differences between elastic predictions of response and inelastic behavior are the same as for the linear static method. Under the linear dynamic procedure, inertial seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic, response spectrum analysis.

The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using modal response spectrum analysis (RSA). Modal spectral analysis is carried out using unreduced, linearly-elastic response spectra scaled to the hazard level (probability of exceedance) inherent in the desired performance objective. As with the LSP, it is expected that the LDP will produce estimates of displacements and interstory drifts that are approximately correct, but will produce estimates of internal forces that exceed those that would be obtained in a yielding building.

3.4.4.2 Analysis

3.4.4.2.1 General

The LDP shall conform to the criteria in Section 3.3.2.2 of *FEMA-273*. The analysis shall be based on appropriate characterization of the ground motion. The requirement that all significant modes be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the building in each of the building's principal horizontal directions. Modal damping ratios should reflect the damping inherent in the building at deformation levels less than the yield deformation. Except for buildings incorporating passive or active energy dissipation devices, or base isolation technology, effective damping shall be taken as 5% of critical.

The interstory drift, and other response parameters calculated for each mode, and required for evaluation in accordance with Section 3.4.4.3, should be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root of the sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

Multidirectional excitation effects may be accounted for by combining 100% of the response due to loading in direction A with 30% of the response due to loading in the direction B; and by combining 30% of the response in direction A with 100% of the response in direction B, where A and B are orthogonal directions of response for the building.

3.4.4.2.2 Ground Motion Characterization

The horizontal ground motion should be characterized by one of the following methods:

1. An elastic response spectrum, developed in accordance with the recommendations of Section 2.6.1.5 in *FEMA-273* for the hazard level contained in the desired performance objective.
2. A site-specific response spectrum developed in accordance with the recommendations of Section 2.6.2.1 of *FEMA-273* for the appropriate hazard level contained in the desired performance objective.

3.4.4.3 Determination of Actions and Deformations

3.4.4.3.1 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the interstory drift results of the response spectrum analysis by the product of the modification factors, C_1, C_2, and C_3 defined in Section 3.4.3 and by the applicable analytical uncertainty factor g_a and demand variability factor g obtained from Section 3.6.2.

3.4.4.3.2 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces, as given in Section 3.4.3.3.6, by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 3.6.3.

3.4.5 Nonlinear Static Procedure

3.4.5.1 Basis of the Procedure

Under the nonlinear static procedure (NSP), a model directly incorporating the inelastic material and nonlinear geometric response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to a pattern of monotonically increased lateral forces or displacements until either a target displacement is exceeded or mathematical instability occurs. The target displacement is intended to approximate the total maximum displacement likely to be experienced by the actual structure, at the hazard level corresponding to the selected performance objective. The target displacement should be calculated in accordance with the procedure presented in Section 3.3.3.3A of *FEMA-273* with modifications, as indicated below. Because the mathematical model accounts directly for effects of material and geometric nonlinear response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake, presuming that an appropriate pattern of loading has been applied.

Interstory drifts and column axial demands obtained from the NSP are evaluated using the applicable procedures of Section 3.6. Calculated interstory drifts, column forces, and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

Commentary: The nonlinear static analysis approach inherently assumes that behavior is dominated by the first mode response of the structure. For this reason, this approach should be used only for structures with relatively short periods. What constitutes a building with a "short period" is dependent on the spectral characteristics of ground shaking anticipated at the site. The small magnitude events, that dominate the hazard at many central and eastern United States sites, tend to have most of their energy at very short periods, particularly on firm soil and rock sites. For sites subject to such shaking, nonlinear static analyses may be valid only for very short and rigid structures. The limitations on the use of NSP, based on period, contained in Table 3-3, are based on recent work that indicates that higher mode response does not tend to become significant in structures responding to ground shaking with typical response spectra unless the fundamental period of the structure is more than about 3.5 times the period at which the spectrum transitions from a range of constant acceleration response to constant velocity response.

A second potential limitation of this procedure is that in practice, two-dimensional models are often used to simulate three-dimensional response. Estimates of load distribution between the lateral-load-resisting elements in the building are required, and the accuracy of the analysis depends upon the accuracy of distribution. Three-dimensional linearly elastic models may be used to estimate this distribution; however, these models are unable to account for load redistribution occurring because of inelastic behavior. When many plastic hinges form nearly simultaneously, creating local frame mechanisms, the effects of torsional contributions may not be accurately represented. If a structure has significant torsional irregularity, three-dimensional models should be used.

The NSP is also limited with regard to evaluation of simultaneous response to ground shaking in different directions. Little research has been performed on appropriate methods of accounting for multi-directional response using this technique. Therefore, these criteria have adapted standard approaches used in linear analysis for this purpose.

3.4.5.2 Analysis Considerations

3.4.5.2.1 General

In the context of these *Recommended Criteria*, the NSP involves the application of incrementally adjusted, monotonically increasing lateral forces, or displacements, to a mathematical nonlinear model of a building, until the displacement of a control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations obtained from both directions used for design.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging between zero and 150% of the target displacement d_t given by Equation 3-11 of *FEMA-273*. Performance evaluation shall be based on those column forces and interstory drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement d_t corresponding to the hazard level (probability of exceedance) appropriate to the performance objective being evaluated.

Gravity loads shall be applied to appropriate elements and components of the mathematical model during the NSP. The loads and load combinations shall be as follows:

1. 100% of computed dead loads and permanent live loads shall be applied to the model.
2. 25% of transient floor live loads shall be applied to the model, except in warehouse and storage occupancies, where the percentage of live load used in the analysis shall be based on a realistic assessment of the average long-term loading.

The analysis model should be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

Commentary: As with any nonlinear model, the ability of the analyst to detect the presence of inelastic behavior requires the use of a nonlinear finite element at the assumed location of yielding. The model will fail to detect inelastic behavior when appropriately distributed finite elements are not used. However, as an alternative to the use of nonlinear elements, it is possible to use linear elements and reconfigure the model, for example, by adjusting member restraints, as nonlinearity is predicted to occur. For example, when a member is predicted to develop a plastic hinge, a linear model can be revised to place a hinge at this location. When this approach is used, the internal forces and stresses that caused the hinging must be reapplied as a nonvarying static load.

The recommendation to continue the pushover analysis to displacements that are 150% of the target displacement is to allow an understanding of the probable behavior of the building under somewhat larger loading than anticipated. If the pushover analysis should become unstable prior to reaching 150% of the target displacement, this does not indicate that a design is unacceptable, but does provide an indication of how much reserve remains in the structure at the design ground motion.

3.4.5.2.2 Control Node

The NSP requires definition of a control node in the building. These *Recommended Criteria* consider the control node to be the center of mass at the roof of the building; the top of a penthouse should not be considered as the roof unless it is of such substantial area and construction as to materially affect the response. The displacement of the control node is compared with the target displacement – a displacement that characterizes the effects of earthquake shaking at the desired hazard level.

3.4.5.2.3 Lateral Load Patterns

Lateral loads should be applied to the building in profiles given in Section 3.3.3.2C of *FEMA-273*.

3.4.5.2.4 Period Determination

The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the NSP as described in Section 3.3.3.2D of *FEMA-273*.

3.4.5.2.5 Analysis of Three-Dimensional Models

Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level.

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required by Section 3.2.7 in *FEMA-273*. Refer also to Section 3.4.5.3.4 of these *Recommended Criteria*.

3.4.5.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) of the building should be developed for two-dimensional analysis. The effects of horizontal torsion should be considered as required by Section 3.2.2.2 of *FEMA-273*.

3.4.5.2.7 Connection Modeling

Existing, fully restrained, unimproved welded moment-resisting connections should be modeled as indicated in Section 6.2.1.2 of these *Recommended Criteria*. Simple shear tab connections with slabs present should be modeled as indicated in Section 6.2.2.1.2. Improved or upgraded fully restrained moment-resisting connections should be modeled as for unimproved connections except that the quantity q_{SD} should be as indicated in Chapter 6 for the applicable connection type.

3.4.5.3 Determination of Actions and Deformations

3.4.5.3.1 Target Displacement

The target displacement, d_t, for buildings with rigid diaphragms at each floor level shall be estimated using the procedures of Section 3.3.3.3A of FEMA-273. Actions and deformations corresponding to the control node displacement equal to the target displacement shall be used for performance evaluation in accordance with Section 3.6.

3.4.5.3.2 Diaphragms

The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

3.4.5.3.3 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum interstory drift calculated at the target displacement by the applicable analytical uncertainty factor g_a and demand variability factor g obtained from Section 3.6.2.

3.4.5.3.4 Multidirectional Effects

Multidirectional excitation effects may be accounted for by combining 100% of the response due to loading in direction A with 30% of the response due to loading in the direction B; and by combining 30% of the response in direction A with 100% of the response in direction B, where A and B are orthogonal directions of response for the building.

An acceptable alternative to this approach is to perform the nonlinear static analysis simultaneously in two orthogonal directions by application of 100% of the loading in direction A simultaneously with 30% of the loading in direction B. Loading shall be applied until 100% of the target displacement in direction A is achieved. This procedure shall be repeated with 30% of the loading applied in direction A and 100% in direction B.

3.4.5.3.5 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces at the target displacement by the applicable analytical uncertainty factor g_a and demand variability factor, g, from Section 3.6.3.

3.4.6 Nonlinear Dynamic Procedure

3.4.6.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), inertial seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis.

The basis, the modeling approaches, and the acceptance criteria for the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using response-history analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using suites of ground motion records. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is necessary to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material and geometric inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 3.6. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values for the applicable performance level.

3.4.6.2 Analysis Assumptions

3.4.6.2.1 General

The NDP shall conform to the criteria given in Section 3.3.4.2A of *FEMA-273*.

3.4.6.2.2 Ground Motion Characterization

The earthquake shaking should be characterized by suites of ground motion acceleration histories, prepared in accordance with the recommendations of Section 2.6.2 of *FEMA-273* and corresponding to the hazard level appropriate to the desired performance objective. A minimum of three pairs of ground motion records shall be used. Each pair shall consist of two orthogonal components of the ground motion.

Consideration of multidirectional excitation effects required by Section 3.2.7 of *FEMA-273* may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along the horizontal axes of the building.

The effects of torsion should be considered according to Section 3.2.2.2 of *FEMA-273*.

3.4.6.3 Determination of Actions and Deformations

3.4.6.3.1 Response Quantities

Response quantities shall be computed as follows:

1. If less than seven pairs of ground motion records are used to perform the analyses, each response quantity (for example, interstory drift demand or column axial demand) shall be taken as the maximum value obtained from any of the analyses.
2. If seven or more pairs of ground motion records are used to perform the analyses, the median value of each of the response quantities computed from the suite of analyses may be used as the demand. The median value shall be that value exceeded by 50% of the analyses in the suite.

3.4.6.3.2 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum of the interstory drifts calculated in accordance with Section 3.4.6.3.1 by the applicable analytical uncertainty factor g_a and demand variability factor g obtained from Section 3.6.2.

3.4.6.3.3 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the column forces calculated in accordance with Section 3.4.6.3.1 by the applicable analytical uncertainty factor g_a, and demand variability factor g from Section 3.6.3 or 3.6.4.

3.5 Mathematical Modeling

3.5.1 Basic Assumptions

In general, a steel moment-frame structure should be modeled and analyzed as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate response information for regular, symmetric structures and structures with flexible diaphragms, three-dimensional mathematical models should be used for analysis and design of buildings with plan irregularity as defined in *FEMA-302*. Two-dimensional modeling, analysis, and design of buildings with stiff or rigid diaphragms are acceptable, if torsional effects are either sufficiently small to be ignored, or are captured indirectly.

Vertical lines of framing in buildings with flexible diaphragms may be individually modeled, analyzed and designed as two-dimensional assemblies of components and elements, or a three-dimensional model may be used, with the diaphragms modeled as flexible elements.

Explicit modeling of connection force-deformation behavior is not required for linear analysis procedures. In nonlinear procedures explicit modeling of connection stiffness is recommended for those cases when the connection is weaker than the connected components, or when it is appropriate to model strength degradation in the connection as a function of imposed deformation demand.

> *Commentary: A finite element model will only collect information at places in the structure where a modeling element is inserted. When nonlinear deformations are expected in a structure, the analyst must anticipate the location of these deformations (such as plastic hinges) and insert nonlinear finite elements at these locations if the inelastic behavior is to be captured by the model.*

3.5.2 Frame Configuration

The analytical model should accurately account for the stiffness of frame elements and connections. Element and component stiffness properties, strength estimates and locations of plastic hinge formation for both linear and nonlinear procedures can be determined from information given in Chapter 6 for typical connections.

3.5.2.1 Elements Modeled

Only the beams and columns forming the lateral-force-resisting system need be modeled, although it shall be permissible to model nonparticipating elements of the structure if realistic assumptions are made with regard to their stiffness, strength and deformation capacity. Refer to Chapter 6 for procedures for modeling common gravity-load beam-column connections.

> *Commentary: Typically, engineers modeling steel moment-frame buildings neglect the participation of gravity-load-carrying beams and columns that are not intended to be part of the lateral-force-resisting system. Studies conducted in support of the development of these recommendations indicate that these*

connections are capable of contributing non-negligible stiffness through large interstory drift demands. Analyses made with models that neglect the participation of these elements will tend to over-estimate demands on the lateral-force-resisting elements and interstory drift demand on the structure.

While it is permissible to conduct performance evaluations using models that neglect non-participating framing, models that include the stiffness of these elements can be used to provide improved levels of confidence with regard to the building's ability to meet desired performance objectives. This is an example of the process by which confidence can be improved, by performing more intense study to reduce the inherent uncertainty.

3.5.2.2 Panel Zone Stiffness

It shall be permissible for the model to assume centerline-to-centerline dimensions for the purpose of calculating stiffness of beams and columns. Alternatively, more realistic assumptions that account for the flexibility of panel zones may be used. Regardless, calculation of moments and shears should be performed at the face of the column.

Commentary: Models that use centerline-to-centerline dimensions for calculation of beam and column stiffness tend to estimate conservatively the interstory drift demand on the structure. While it is permissible to conduct performance evaluations using models that neglect the effect of the panel zone on beam and column stiffness, models that include more realistic estimation of this effect can be used to provide improved levels of confidence with regard to the building's ability to meet desired performance objectives.

A number of models are available to represent panel zones of moment-resisting connections. These range from simple models that idealize the panel zone as a scissors-type model that accounts explicitly for the shear stiffness of the panel zone, and to complex multi-element models that accounts both for shear stiffness of the panel zone and the effects of geometric distortion of the zone. Analyses of buildings using these various models reported in FEMA-355C indicate that the particular model used has relatively little impact on the predicted interstory drift demand. However, for nonlinear analysis models, the element selected to represent the panel zone can have significant impact on where plasticity in the structure is predicted to occur, e.g. in the panel zone itself, within the beam, or a combination of these regions.

3.5.3 Horizontal Torsion

The effects of actual horizontal torsion must be considered. In *FEMA-302*, the total torsional moment at a given floor level includes the following two torsional moments:

1. the actual torsion, that is, the moment resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

2. the accidental torsion, that is, an accidental torsional moment produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

For the purposes of performance evaluation, under these *Recommended Criteria*, accidental torsion should not be considered. In buildings with diaphragms that are not flexible, the effect of actual torsion should be considered if the maximum lateral displacement d_{max} from this effect at any point on any floor diaphragm exceeds the average displacement d_{avg} by more than 10%.

Commentary: Accidental torsion is an artificial device used by the building codes to account for actual torsion that can occur, but is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation. Such torsion can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner in the structure. For example if the frames on the east and west sides of a structure have similar elastic stiffness the structure may not have significant torsion during elastic response. However, if the frame on the east side of the structure yields significantly sooner than the framing on the west side, then inelastic torsion will develop. Rather than requiring that an accidental torsion be applied in the analysis, as do the building codes, these Recommended Criteria indirectly account for the uncertainty related to these torsional effects in the calculation of demand and resistance factors.

3.5.4 Foundation Modeling

In general, foundations may be modeled as unyielding. Assumptions with regard to the extent of fixity against rotation provided at the base of columns should realistically account for the relative rigidities of the frame and foundation system, including soil compliance effects, and the detailing of the column base connections. For purposes of determining building period and dynamic properties, soil-structure interaction may be modeled as permitted by the building code.

Commentary: Most steel moment frames can be adequately modeled by assuming that the foundation provides rigid support for vertical loads. However, the flexibility of foundation systems (and the attachment of columns to those systems) can significantly alter the flexural stiffness at the base of the frame. Where relevant, these factors should be considered in developing the analytical model.

3.5.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic-force-resisting system. Connections between the edge beams of floor and roof diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for evaluation of diaphragm components are given in Section 3.3 of *FEMA-273*.

Development of the mathematical model should reflect the stiffness of the diaphragm. As a general rule, most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms and floors or roofs with plywood diaphragms should be considered flexible. The flexibility of unfilled metal deck, and concrete slab diaphragms with large openings should be considered in the analytical model.

Mathematical models of buildings with diaphragms that are not rigid should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

3.5.6 *P-D* Effects

P-D effects, caused by gravity loads acting on the displaced configuration of the building, may be critical in the seismic performance of steel moment-frame buildings, which are usually flexible and may be subjected to large lateral displacements.

The structure should be investigated to ensure that lateral drifts induced by earthquake response do not result in a condition of instability under gravity loads. At each story, the quantity Y_i should be calculated for each direction of response, as follows:

$$Y_i = \frac{P_i d_i}{V_{yi} h_i} \tag{3–5}$$

where:

P_i = portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on all of the columns within story level *i*,

V_{yi} = total plastic lateral shear force in the direction under consideration at story *i*,

h_i = height of story *i*, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference, and

δ_i = lateral interstory drift in story *i*, from the analysis in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i.

In any story in which Y_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity Y_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by *P-D* effects. When

Ψ_i in a story exceeds 0.3, the structure shall be considered unstable, unless a detailed global stability capacity evaluation for the structure, considering *P-D* effects, is conducted in accordance with the guidelines of Appendix A.

For nonlinear procedures, second-order effects should be considered directly in the analysis; the geometric stiffness of all elements and components subjected to axial forces should be included in the mathematical model.

Commentary: The values of interstory drift capacity for the Collapse Prevention performance level, provided in Section 3.6, and the corresponding resistance factors, were computed considering P-D effects (FEMA-355F). For a given structure, it is believed that if the value of Ψ is less than 0.3 the effects of P-D have been adequately considered by these general procedures. For values of Ψ greater than this limit the statistics on frame interstory drift capacities contained in Section 3.6 are inappropriate. For such frames explicit determination of interstory drift capacities, considering P-D effects using the detailed Performance Evaluation procedures outlined in Appendix A is required.

The plastic story-shear quantity, V_{yi} should be determined by methods of plastic analysis. In a story in which: (1) all beam-column connections meet the strong-column-weak-beam criterion, (2) the same number of moment-resisting bays is present at the top and bottom of the frame, and (3) the strength of moment-connected girders at the top and bottom of the frame is similar, V_{yi} may be approximately calculated from the equation:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG\,j}}{h_i} \tag{3-6}$$

where:

M_{pGj} = *the expected plastic moment capacity of each girder "j" participating in the moment resisting framing at the floor level on top of the story*

n = *the number of moment-resisting girders in the framing at the floor level on top of the story*

In any story in which none of the columns meets the strong-column-weak-beam criterion, the plastic story-shear quantity, V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{n} M_{pC_k}}{h_i} \tag{3-7}$$

where:

M_{pCk} = *the plastic moment capacity of each column "k", participating in the moment resisting framing, considering the axial load present on the column.*

For other conditions, the quantity V_{yi} must be calculated by plastic mechanism analysis, considering the vertical distribution of lateral forces on the structure.

3.5.7 Multidirectional Excitation Effects

Buildings should be evaluated for response due to seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of a building. For buildings with plan irregularity and buildings in which one or more components form part of two or more intersecting elements, multidirectional excitation effects should be considered, as indicated in Section 3.4 for the various analytical procedures.

3.5.8 Vertical Ground Motion

The effects of vertical excitation on horizontal cantilevers may be considered either by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 2/3 of those of the horizontal spectrum unless alternative vertical response spectra are developed using site-specific analysis. Vertical earthquake effects on other beam elements and column elements need not be considered.

Commentary: There is no evidence that response to vertical components of ground shaking has had any significant effect on the performance of steel moment-frame buildings. Consequently, the effect of this response is not recommended for consideration in the performance evaluation, except as required by the building code for the case of horizontal cantilever elements.

Traditionally, vertical response spectra, when considered, have been taken as 2/3 of the horizontal spectra developed for the site. While this is a reasonable approximation for most sites, vertical response spectra at near-field sites, located within a few kilometers of the zone of fault rupture, can have substantially stronger vertical response spectra than indicated by this approximation. Development of site-specific response spectra is recommended when vertical response must be considered for buildings on such sites.

3.6 Acceptance Criteria

Acceptability of building performance should be evaluated by determining a level of confidence in the building's ability to meet the desired performance objective(s). The parameters in Table 3-5 must be independently evaluated, using the procedures of Section 3.6.1 and the parameters and acceptance criteria of Sections 3.6.2, 3.6.3, and 3.6.4, for each performance objective evaluated. The controlling parameter is that which results in the calculation of the lowest confidence for building performance.

Table 3-5 Performance Parameters Requiring Evaluation of Confidence

Parameter	Discussion
Interstory drift	The maximum interstory drift computed for any story of the structure shall be evaluated for global and local behavior (for Collapse Prevention and Immediate Occupancy). Refer to Section 3.6.2
Column axial load	The adequacy of each column to withstand its calculated maximum compressive demand shall be evaluated both for Collapse Prevention and Immediate Occupancy. Refer to Section 3.6.3
Column splice tension	The adequacy of column splices to withstand their calculated maximum tensile demands shall be evaluated both for Collapse Prevention and Immediate Occupancy. Refer to Section 3.6.4

3.6.1 Factored-Demand-to-Capacity Ratio

Confidence level is determined through evaluation of the factored-demand-to-capacity ratio given by the equation:

$$\lambda = \frac{\gamma \gamma_a D}{\phi C} \tag{3-8}$$

where:

C = capacity of the structure, as indicated in Sections 3.6.2, 3.6.3, and 3.6.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

D = calculated demand for the structure, obtained from structural analysis.

γ = a demand variability factor that accounts for the variability inherent in the prediction of demand related to assumptions made in structural modeling and prediction of the character of ground shaking as indicated in Sections 3.6.2, 3.6.3, and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

γ_a = an analytical uncertainty factor that accounts for bias and uncertainty, inherent in the specific analytical procedure used to estimate demand as a function of ground shaking intensity, as indicated in Section 3.6.2, 3.6.3 and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

ϕ = a resistance factor that accounts for the uncertainty and variability inherent in the prediction of structural capacity as a function of ground shaking intensity, as indicated in Section 3.6.2, 3.6.3 and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

λ = a confidence index parameter from which a level of confidence can be obtained. See Table 3-6.

Factored-demand-to-capacity ratio l shall be calculated using equation 3-8 for each of the performance parameters indicated in Table 3-5, which also references the appropriate section of these *Recommended Criteria* where the various parameters, g_a, g, f required to perform this evaluation may be found. These referenced sections also define an uncertainty parameter b_{UT} associated with the evaluation of global and local interstory drift capacity, column compressive capacity, and column splice tensile capacity, respectively. These uncertainties are related to the building's configuration, the type of moment-resisting connections present (type 1 or type 2), the type of analytical procedure employed and the performance level being evaluated. Table 3-6 indicates the level of confidence associated with various values of the factored-demand-to-capacity ratio l calculated using Equation 3-8, for various values of the uncertainty parameter b_{UT}. Linear interpolation between the values given in Table 3-6 may be used for values of factored-demand-to-capacity ratio l and uncertainty b_{UT} intermediate to those tabulated.

Table 3-6 Factored-Demand-to-Capacity Ratios l for Specific Confidence Levels and Uncertainty b_{UT} factors

	Confidence Level										
Uncertainty Parameter b_{UT}	**10**	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**95**	**99**
0.2	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
0.3	1.68	1.48	1.34	1.23	1.14	1.06	0.98	0.89	0.78	0.70	0.57
0.4	2.12	1.79	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.51
0.5	2.76	2.23	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
0.6	3.70	2.86	2.36	1.99	1.72	1.48	1.25	1.03	0.80	0.64	0.43

Table 3-7 provides minimum recommended levels of confidence for each of the potential controlling behavior modes, that is, global stability, local connection capacity, column buckling or column splice tensile failure, for the Immediate Occupancy and Collapse Prevention performance levels, respectively.

Table 3-7 Recommended Minimum Confidence Levels

Behavior	Performance Level	
	Immediate Occupancy	Collapse Prevention
Global Behavior Limited by Interstory Drift	50%	90%
Local Connection Behavior Limited by Interstory Drift	50%	50%
Column Compression Behavior	50%	90%
Column Splice Tension Behavior	50%	50%

Commentary: In order to predict structural performance, these procedures rely on the application of structural analysis and laboratory test data to predict the behavior of real structures. However, there are a number of sources of uncertainty inherent in the application of analysis and test data to performance prediction. For example, the actual strength of structural materials, the quality of individual welded joints, and the amount of viscous damping present is never precisely known, but can have impact on both the actual amount of demand produced on the structure and its elements and their capacity to resist these demands. If the actual values of these and other parameters that affect structural performance were known, it would be possible to predict accurately both demand and capacity. However, this is never the case. In these procedures, confidence is used as a measure of the extent that predicted behavior is likely to represent reality.

The extent of confidence inherent in a performance prediction is related to the possible variation in the several factors that affect structural demand and capacity, such as stiffness, damping, connection quality, and the analytical procedures employed. In this project, evaluations were made of the potential distribution of each of these factors and the effect of variation in these factors on structural demand and capacity. Each of these sources of uncertainty in structural demand and capacity prediction were characterized as part of the supporting research for this project, by a coefficient of variation, b_U. The coefficient, b_{UT} is the total coefficient of variation, considering all sources of uncertainty. It is used, together with other factors to calculate the demand and resistance factors. We assume that demand and capacity are lognormally distributed relative to these uncertainty parameters. This allows confidence to be calculated as a function of the number of standard deviations that factored-

demand-to-capacity-ratio l lies above or below a mean value. Table 3-6 provides a solution for this calculation, using a conservative estimate of the hazard parameter, k=3.0, that is representative of the typical seismicity of coastal California. Further information on this method may be found in Appendix A.

3.6.2 Performance Limited by Interstory Drift Angle

3.6.2.1 Factored Interstory Drift Angle Demand

Factored interstory drift demand should be computed as the quantity gg_aD where the demand D, is the largest interstory drift in any story computed from structural analysis, g_a is the coefficient obtained from Table 3-8, and g is the coefficient obtained from Table 3-9.

Table 3-8 Interstory Drift Angle Analysis Uncertainty Factors, g_a

Analysis Procedure	LSP		LDP		NSP		NDP	
System Characteristic	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]
Type 1 Connections								
Low Rise (<4 stories)	0.94	0.70	1.03	0.83	1.13	0.89	1.02	1.03
Mid Rise (4-12 stories)	1.15	0.97	1.14	1.25	1.45	0.99	1.02	1.06
High Rise (> 12 stories)	1.12	1.21	1.21	1.14	1.36	0.95	1.04	1.10
Type 2 Connections								
Low Rise (<4 stories)	0.79	0.98	1.04	1.32	0.95	1.31	1.02	1.03
Mid Rise (4-12 stories)	0.85	1.14	1.10	1.53	1.11	1.42	1.02	1.06
High Rise (> 12 stories)	0.80	0.85	1.39	1.38	1.36	1.53	1.04	1.10

Notes: 1- I.O. = Immediate Occupancy Performance Level
2- C.P. = Collapse Prevention Performance Level

Commentary: Several structural response parameters are used to evaluate structural performance. The primary parameter is interstory drift. Interstory drift is an excellent parameter for judging the ability of a structure to resist P-D instability and collapse. It is also closely related to plastic rotation demand, or drift angle demand, on individual beam-column connection assemblies, and is therefore a good predictor of the performance of beams, columns and connections. For tall slender structures, a significant portion of interstory drift is a result of axial elongation and shortening of different rows of columns. Although modeling of the structure should account for this frame flexibility, that portion of interstory drift resulting from axial column deformation in stories *below the story under consideration should be neglected in determining local connection performance. This portion of the interstory drift must usually be determined manually as most computer programs do not calculate this quantity separately.*

Table 3-9 Interstory Drift Angle Demand Variability Factors g

Building Height	Immediate Occupancy (I.O.)	Collapse Prevention (C.P.)
Type 1 Connections[1]		
Low Rise (3 stories or less)	1.5	1.3
Mid Rise (4-12 stories)	1.4	1.2
High Rise (more than 12 stories)	1.4	1.5
Type 2 Connections[2]		
Low Rise (3 stories or less)	1.4	1.4
Mid Rise (4-12 stories)	1.3	1.5
High Rise (more than 12 stories)	1.6	1.8

Notes:

1- Type 1 connections are capable of resisting median total drift angle demands of 0.04 radians without fracture or strength degradation.
2- Type 2 connections are capable of resisting median total drift angle demands of 0.01 radians without fracture or strength degradation. Generally, welded unreinforced connections, employing weld metal with low notch toughness, typical of older welded steel moment-frame buildings should be considered to be this type.

3.6.2.2 Factored Interstory Drift Angle Capacity

Interstory drift capacity may be limited either by the global response of the structure, or by the local behavior of beam-column connections. Section 3.6.2.2.1 provides values for global interstory drift capacity for regular, well-configured structures as well as associated uncertainties, b_{UT}. Global interstory drift capacities for irregular structures must be determined using the detailed procedures of Appendix A. Section 3.6.2.2.2 provides procedures for evaluating local interstory drift angle capacity, as limited by connection behavior.

3.6.2.2.1 Global Interstory Drift Angle

Factored interstory drift angle capacity, fC, as limited by global response of the building, shall be based on the product of the resistance factor f and capacity C, which are obtained from Table 3-10, for either Type 1 or Type 2 connections. Type 1 connections are capable of resisting median total interstory drift angle demands of 0.04 radians without fracturing or strength degradation. Type 2 connections are capable of resisting median total interstory drift angle demands of 0.01 radian without fracturing or strength degradation. Welded unreinforced moment-resisting connections with weld metal with low notch toughness should be considered Type 2. Table 3-11 provides values of the uncertainty coefficient b_{UT} to be used with global interstory drift evaluation.

Table 3-10 Global Interstory Drift Angle Capacity *C* and Resistance Factors *f* for Regular Buildings

Building Height	Performance Level			
	Immediate Occupancy		Collapse Prevention	
	Interstory Drift Angle Capacity *C*	Resistance Factor *f*	Interstory Drift Angle Capacity *C*	Resistance Factor *f*
Type 1 Connections				
Low Rise (3 stories or less)	0.02	1.0	0.10	0.90
Mid Rise (4 – 12 stories)	0.02	1.0	0.10	0.85
High Rise (> 12 stories)	0.02	1.0	0.085	0.75
Type 2 Connections				
Low Rise (3 stories or less)	0.01	1.0	0.10	0.85
Mid Rise (4 – 12 stories)	0.01	0.9	0.08	0.70
High Rise (> 12 stories)	0.01	0.85	0.06	0.60

3.6.2.2.2 Local Interstory Drift Angle

Factored interstory drift angle fC limited by local connection response, shall be based on the capacity C of the connection and resistance factor f obtained from Chapter 6 of these *Recommended Criteria*. For Immediate Occupancy performance, capacity C shall be taken as the quantity q_{IO} and for Collapse Prevention performance, the quantity q_U indicated in Chapter 6 for the connection types present in the building. For connection types not include in Chapter 6, the capacity and resistance factors should be obtained from laboratory testing and the procedures of Appendix A. Table 3-12 provides values of the uncertainty coefficient b_{UT} for use in evaluating performance as limited by local connection behavior.

Table 3-11 Uncertainty Coefficient b_{UT} for Global Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Type 1 Connections		
Low Rise (3 stories or less)	0.20	0.3
Mid Rise (4 – 12 stories)	0.20	0.4
High Rise (> 12 stories)	0.20	0.5
Type 2 Connections		
Low Rise (3 stories or less)	0.20	0.35
Mid Rise (4 – 12 stories)	0.20	0.45
High Rise (> 12 stories)	0.20	0.55

Notes: 1- Value of b_{UT} should be increased by 0.05 for LSP analysis
2- Value of b_{UT} may be reduced by 0.05 for NDP analysis

Table 3-12 Uncertainty Coefficient b_{UT} for Local Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Type 1 Connections		
Low Rise (3 stories or less)	0.30	0.30
Mid Rise (4 – 12 stories)	0.30	0.35
High Rise (> 12 stories)	0.30	0.40
Type 2 Connections		
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.30	0.40
High Rise (> 12 stories)	0.30	0.40

Notes: 1- Value of b_{UT} should be increased by 0.05 for LSP analysis
2- Value of b_{UT} may be reduced by 0.05 for NDP analysis

3.6.3 Performance Limited by Column Compressive Capacity

3.6.3.1 Column Compressive Demand

Factored column compressive demand shall be determined for each column as the quantity gg_aD, where:

D = the compressive axial load on the column determined as the sum of Dead Load, 25% of unreduced Live Load, and Seismic Demand. Seismic demand shall be determined by one of the following four analysis methods:

Linear: The axial demands may be taken as those predicted by a linear static or linear dynamic analysis, conducted in accordance with Section 3.4.3 or 3.4.4.

Plastic: The axial seismic demands may be taken from plastic analysis, as indicated by Equation 3-5 in Section 3.4.3.3.6.

Nonlinear Static: The axial demands may be taken from the computed forces from a nonlinear static analysis, at the target displacement, in accordance with Section 3.4.5.

Nonlinear Dynamic: The axial demands may be taken from the computed design forces from a nonlinear dynamic analysis, in accordance with Section 3.4.6.

g_a = analytical uncertainty factor, taken from Table 3-13.

g = demand variability demand factor, taken as having a value of 1.05.

The uncertainty coefficient b_{UT} shall be taken as indicated in Table 3-13 based on the procedure used to calculate column compressive demand *D*.

Table 3-13 Analysis Uncertainty Factor g_a and Total Uncertainty Coefficient b_{UT} for Evaluation of Column Compressive Demands

Analytical Procedure	Analysis Uncertainty Factor g_a	Total Uncertainty Coefficient b_{UT}
Linear Static or Dynamic Analysis	1.15	0.35
Plastic Analysis (Section 3.4.3.3.6)	1.0	0.15
Nonlinear Static Analysis	1.05	0.20
Nonlinear Dynamic Analysis	$e^{1.4b^2}$	$\sqrt{0.0225+b^2}$

Note: *b* may be taken as the coefficient of variation (COV) of the axial load values determined from the suite of nonlinear analyses

Commentary: The value of g has been computed assuming a coefficient of variation for axial load values resulting from material strength variation and uncertainty in dead and live loads of 15%. The values of g_a have been calculated assuming coefficients of variation of 30%, 0% and 15%, related to uncertainty in the analysis procedures for linear, plastic and nonlinear static analyses, respectively. In reality, for structures that are stressed into the inelastic range, elastic analysis will typically overestimate axial column demands, in which case, a value of 1.0 could be used. However, for structures that are not loaded into the inelastic range, the indicated value is appropriate. Plastic analysis will also *typically result in an upper bound estimate of column demand and application of additional demand factors is not appropriate. For nonlinear dynamic analysis, using a suite of ground motions, direct calculation of the analysis demand factor is possible, using the equation shown. All of these demand factors are based on the hazard parameter k having a value of 3, which is typical of conditions in coastal California.*

3.6.3.2 Column Compressive Capacity

Factored compressive capacity of each individual column to resist compressive axial loads shall be determined as the product of the resistance factor, f, and the nominal axial strength C of the column, which shall be determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, for the purposes of this evaluation, the effective length coefficient k shall be taken as having a value of 1.0 and the resistance factor f shall be assigned a value of 0.95.

3.6.4 Column Splice Capacity

The capacity of column tensile splices, other than splices consisting of complete joint penetration (CJP) butt welds of all elements of the column (flanges and webs) shall be evaluated in accordance with this section. Column splices consisting of CJP welds of all elements of the column, and in which the weld filler metal conforms to the recommendations of Sections 6.4.2.4 and 6.4.2.5 of these *Recommended Criteria* need not be evaluated.

3.6.4.1 Column Splice Tensile Demand

Factored column splice tensile demand shall be determined for each column as the quantity gg_aD where D is the column splice tensile demand. Column splice tensile demand shall be determined as the computed Seismic Demand in the column less 90% of the computed Dead Load demand. Seismic Demand shall be as determined for column compressive demand, in accordance with Section 3.6.3.1. The demand variability factor g shall be taken as having a value of 1.05 and the analysis uncertainty factor g_a shall be taken as indicated in Table 3-13. The total uncertainty coefficient $\boldsymbol{b}_{UT}$ shall also be taken as indicated in Table 3-13.

3.6.4.2 Column Splice Tensile Capacity

The capacity of individual column splices to resist tensile axial loads shall be determined as the product of the resistance factor, f, and the nominal tensile strength of the splice, C, as determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, Chapter J of the *AISC Specification* shall be used to calculate the nominal tensile strength of the splice connection. For the purposes of this evaluation, f shall be assigned a value of 0.9.